MEMORANDUM TO: W. B. Drake
Assistant State Highway Engineer
Research and Development

DATE: December 17, 1969

SUBJECT: I64 - 6(15) 130, Rowan County
I64 - 6(7) 109, Montgomery - Bath Counties

REFERENCES:
1. BPR letter, February 27, 1969
2. Mr. Kemper's letter to BPR, March 21, 1969
3. BPR reply, April 1, 1969
4. Your memo to me, May 1, 1969
5. Your letter to BPR, July 9, 1969
6. BPR reply, July 17, 1969

First, your request *(4)* that we submit a research proposal to determine the cause of low strengths reported from the Rowan County project seemed preclusive. Only a fact-finding inquiry seemed prerequisite. Mr. Kemper's letter *(2)* and memorandum (April 23) indicated that all possibilities had been explored from the standpoint of construction controls. The Bureau *(1)* also conducted a preliminary investigation and found no assignable cause for the low strengths. It seemed compelling, therefore, to proceed directly into the research in an exploratory way. Second, it seemed necessary to identify and isolate the controlling variable.

To avoid possible confusion here, I shall recapitulate and define the problem. The 28-day strengths required of pavement concrete are: 3500 psi (molded cylinder) and 550 psi (modulus of rupture, molded beam). Although beam specimens may be molded and tested earlier than 28 days, they are not considered to be acceptance-type tests. Cores are taken to check the thickness of the pavement. Sometime ago, it seemed worthwhile -- as a matter of information -- to also test the cores for strength. The Materials Division made this a regular practice. Somehow, in the case of the Bath-Montgomery County project (I64-6(7)-109), undue suspicion was aroused by the Pavement Core Drill Report; the age of the concrete at the time the strength test was made is shown there; but apparently the fact that many specimens were tested earlier than 28 days was overlooked. Even so, there is nothing that requires the pavement concrete to equal cylinder or beam strengths in 28 days. The use of the term "failed" in referring to beam tests made according to Article 307.3.17 (Opening of Pavement) is also subject to misinterpretation.

I must digress momentarily to relate some pertinent chronological details. The Montgomery-Bath County project was constructed in 1967. Apparently the Certification of Materials was not submitted to the Bureau until March 27, 1969. Apparently, too, the Pavement Core Drill Report (March 20, 1968) was submitted as part of the Certification of Materials. The Bureau responded (April 25, 1969) with an expression of concern and alluded to a promise or commitment of continued investigation. The Bureau's letter of February 27, 1969 *(1)*, referred only to the Rowan County Project and specifically to the Core Drill Report dated December 17, 1968. The paving in Rowan County was completed September 27, 1968. Then on July 17, 1969 *(6)*, the Bureau requested that the investigations of the Rowan County project be extended to include the Montgomery-Bath County project. At first, this seemed to compound the assignment too much; however, as previously stated, the Montgomery-Bath matter was easily resolved. Through the assistance of Dan C. Woodward (Division of Construction) and W. H. Baker (Division of Materials), various data and available information were reviewed. Baker noted the discrepancy in the age of the cores, and I urged him to analyze the matter fully and to report independently. He has, by memorandum dated October 13, 1969, addressed to me and Mr. Woodward as members of the task force, resolved all discrepancies in a straightforward, convincing way. His report is included herewith; in my opinion it concludes the investigation of the Montgomery-Bath project. His recommendations are intended to avoid recurrences of an unfortunate misunderstanding.

I may now direct your attention to the Rowan County project. All beams were tested at 7 to 14 days -- pursuant to Article 307.3.17 (Opening of Pavements). Article 307.3.4-D alludes to 28-day strengths for both beams and cylinders. Here again, use of the term "failed" is misleading; it actually means that the beams did not achieve the 28-day, required strength in 7 to 14 days. The majority of them did equal or exceed the required 28-day strength; a few did not. Nevertheless, the beam-strength test results attest to the quality of concrete batched on the project. In other words, the beam tests alone suffice to dispel all suspicions concerning deficient proportioning or batching of concrete on the project. Indeed, the 28-day cylinder strengths were predominantly adequate. There was not a single day when an adequate cylinder strength was not obtained. Only in one instance are consecutive or "paired" failures noted. Thus, each instance of failure may be "nullified" in some degree by a beam strength or an intervening cylinder strength test. Therefore, except for spurious data, it may be reasonably concluded that the concrete produced was capable of achieving the required strength. This conclusion is an important one from the standpoint of the analysis of core strengths.

The first cores from the Rowan County project were taken between October 23 and 31, 1968, and tested December 4, 1968; not a single core yielded 3500 psi. Check cores, tested on February 24, 1969, showed only slight gains in strength in a few instances (9 out of 60 exceeded 3500 psi); 31 out of 60 gave strengths which were less than the original core strengths. A third set of cores was taken September 23, 1969, and tested between September 26 and October 3, 1969. Fortunately remnants of the first check cores had been preserved in the central laboratory. By inspection, several appeared to contain large voids. It was deemed advisable to examine them from the standpoint of unit weight (density). Both SSD and OD densities were determined. The densities of the third set of cores were measured before strength tests were made. These cores were taken at five strategic stations, in pairs, and at three points transversely across the pavement. In a general way, it appeared that those which yielded lower strengths also had lower densities. No other trend was found. In confirmation of the density tests, voids were measured on polished sections of remnants of cores -- by the linear traverse method.

In summary, it should be mentioned again that in all likelihood the cylinder and beam strengths would not have aroused suspicion -- that is, in applying the "reasonable conformity rule". The uncertainty, therefore, is concerned only with the core strengths -- that is, strength of the pavement.

Moreover, it should be borne in mind that cylinder and beam specimens are consolidated by hand -- which may not be comparable to the consolidation of the concrete *in situ.* Whereas hand consolidation of specimens is usually assumed to be adequate, there is no confirming test or measure to show that the pavement concrete was adequately consolidated.

The paving train consisted of a Maxon Spreader, a Rex Spreader, Mesh Bridge, Mesh Placer, a Rex Finisher (2 screeds, 1 pan), a Rex Belt (with 2 burlap drags attached), and a Rex Spray (for curing membrane). The mesh depressor had been altered by the contractor. There was some controversy about spreading; apparently at the beginning, the concrete was deposited ahead in a windrow along one side only; later, it was deposited more-or-less equally in each lane. Only two internal vibrators were used; these were located 18 inches from each edge.

A "rule-of-thumb" check on the amount of agitation and (or) vibration needed to consolidate concrete is to obtain a "frothy" laitance (indicating dissipation of entrained air) and then to stop just short of that condition. Any entrapped air would tend to dissipate before the froth appears. Densities might be measured (with nuclear gages) behind the finishing machines -- a difference of 1.5 pounds in the density would be equivalent to about 1% voids. It would be necessary, of course, to operate a nuclear gage with considerable precision in order to obtain meaningful results.

In regard to the control of air contents, it should not be permissive to use the upper limits of the specification to compensate for normal losses in volume of concrete -- losses of 1% or thereabout may be expected because of waste and shrinkage.

Excess voidage should be considered to be detrimental to strength and to durability. The greater absorptivity may cause premature disintegration by freezing and thawing. The low compressive strengths may lead to "blowups". For example, thermal stresses alone may be in the order of 1800 to 2400 psi; concrete having a nominal strength of 3000 psi is more likely to fatigue than concrete having strengths in the order of 4400 psi. In the event of an epidemic of blowups, relief joints might prove to be warranted.

No further investigation of either project is anticipated.

Respectfully submitted,

Jas. H. Havens
Director of Research

Enclosures

1. Research Report; I64-6(15)130, Rowan County
2. Memorandum Report; Division of Materials; I64-6(7)109, Bath-Montgomery Counties

ccs: A. O. Neiser
F. G. Kemper
Ralph Waddle
J. E. McChord
W. H. Baker

Research Report

AN INVESTIGATION OF CORE-STRENGTHS OF A PORTLAND CEMENT CONCRETE PAVEMENT

[I64 - 6(15)130, Rowan County]

KYP-56

by
Jas. H. Havens
Director

Division of Research
DEPARTMENT OF HIGHWAYS
Commonwealth of Kentucky

December 1969

INTRODUCTION

This investigation concerns observed differences between the strengths of molded cylinder and beam specimens and the strength of concrete *in situ.* The specific project is a section of I 64 in Rowan County [I 64-6(15)130]. The paving was done between August 19 and September 27, 1968. Cylinder specimens molded on the project were tested after they had cured 28 days. Beam specimens were tested before they had cured 14 days. Those test results were "normal". However, cores taken to measure thickness of the pavement were subsequently tested for strength; and the pavement cores, even though older when tested, yielded strengths which were considerably below 28-day cylinder strengths.

Having already explored all reasonable courses of investigation and inquiry, the matter was referred to Research.

The findings reported herein indicate that the concrete was manufactured or batched in nearly the proper proportions but was not well consolidated *in situ.* Excess voidages arising from excess entrained and entrapped air suffice to explain the lower strength of the concrete in the pavement.

REVIEW OF PROJECT RECORDS

Cylinder and beam strengths are shown in Appendix I. All beams were tested at 7 to 14 days -- pursuant to Article 307.3.17 (Opening of Pavement). Article 307.3.4-D alludes to 28-day strengths for both beams and cylinders. Here, use of the term "failed" is misleading; it actually means that the beams did not achieve the 28-day, required strength in 7 to 14 days. The majority of them did equal or exceed the required 28-day strength; a few did not. Nevertheless, the beam-strength test results attest to the quality of concrete batched on the project. In other words, the beam tests alone suffice to dispel all suspicions concerning deficient proportioning or batching of concrete on the project. Indeed, the 28-day cylinder strengths were predominantly adequate. There was not a single day when an adequate cylinder strength was not obtained. Only in one instance are consecutive or "paired" failures noted; this occurred September 24; beams cast on that same day demonstrated "adequate" strength -- that is, considering their age at time of test. Thus, each instance of failure may be "nullified" in some degree by a beam strength or an intervening cylinder strength test. Therefore, except for spurious data or within-day variations in proportioning, it may be reasonably concluded that the concrete produced was capable of achieving the required strength. This conclusion is an important one from the standpoint of the analysis of core strengths.

It is presumptive to expect pavement concrete to gain strength at the same rate as cylinders. For instance, the ideal temperature with respect to strength gain is about 70°F. Concrete cured in summer heat does not achieve strengths comparable to that achieved at 70° F. Low temperatures also retard the development of strength. For this reason, the fact that check cores taken and tested in December did not exhibit vast improvements in strengths should not be too surprising.

Moreover, it should be borne in mind that cylinder and beam specimens are consolidated by hand -- which may not be comparable to the consolidation of the concrete *in situ*. Whereas hand consolidation of specimens is usually assumed to be adequate, there is no confirming test or measure to show that the pavement concrete was adequately consolidated.

In summary, it should be mentioned again that in all likelihood the cylinder and beam strengths would not have aroused suspicion -- that is, in applying the "reasonable conformity rule". The uncertainty, therefore, is concerned only with the core strengths -- that is, strength of the pavement.

Cement Check

Shipments of cement to the project were audited with respect to the volume of concrete batched. Fig. 1 shows an accumulative-type graph, according to calendar days. These data indicate that sufficient cement was shipped to the project.

Air Entrainment and Slump

The specifications (307.3.4-C) presently require 6 ± 2 percent air in pavement concrete. Formerly (1956 **Standard Specifications ...**), they required 3 to 6 percent. The change evolved from reductions in the maximum size of coarse aggregates. There was then an underlying notion that air content should be proportional to the percentage of mortar in the concrete and that the use of smaller coarse aggregate would naturally require a greater proportion of mortar. Apparently this has not been borne out by field practices; the proportions of fine aggregate to coarse aggregate has continued constant. Apparently too, the water demand has not increased. While it is possible that increasing the air requirement may have compensated for any deficiencies in slump and workability, the fact remains that the proportions of mortar and water have not increased. Consequently, the higher permissive limit (8%) may have been extended so far that strength *may* have been affected -- especially if the job control exceeds the preferred value (6%).

The records from the Rowan County project (Appendix II, showing slump and air measurements) indicate that 76 out of 91 air measurements were 6% or greater; 31 measurements were 7% or greater. A strong bias toward high air contents is evident.

Here, again, it is important to bear in mind that inadequate consolidation of the concrete *in situ* would cause additional voidage in the pavement concrete. This would further affect strength.

If, in fact, the pavement concrete did contain voids in greater percentages than the entrained air, microscopic examination of cores (by linear traverse) should disclose the excess -- that is, the voids must exceed 8% in each offending case.

ANALYSIS OF PAVEMENT STRENGTHS

Core Chronology and Data

According to field records, paving began August 19, 1968, and ended September 27, 1968. The first cores (to check pavement thickness) were taken between October 23 and 31, 1968. These cores were tested for strength on December 4, 1968; not a single core yielded 3500 psi. A series of check cores were taken, and these were tested on February 24, 1969. A few of the check cores yielded strengths greater than that of the original cores; 9 out of 60 exceeded 3500 psi; 31 out of 60 gave strengths which were less than that of the original cores. A third series of cores were taken September 23, 1969, and were tested between September 26 and October 3, 1969.

The strengths of the original cores and the check cores are shown in Appendix III.

By inspection, several of the check cores appeared to contain large voids. This observation led to further consideration of unit weights (densities) of the pavement concrete. Remnants of 20 of the check cores had been preserved in the central laboratory. It was possible to obtain SSD and OD unit weights from them. These data are given in Table I. The third series of cores were taken in "pairs"; one of each pair was sawed in two -- giving a top and bottom. Three pairs were taken at five selected stations; one pair in each set was taken 18 inches from the outside edge of the pavement; one was taken near the centerline; and the third was taken 18 inches from the median edge. The data pertaining to this series of cores are also presented in Table I. Only the SSD unit weights and strengths of the "O" cores were determined.

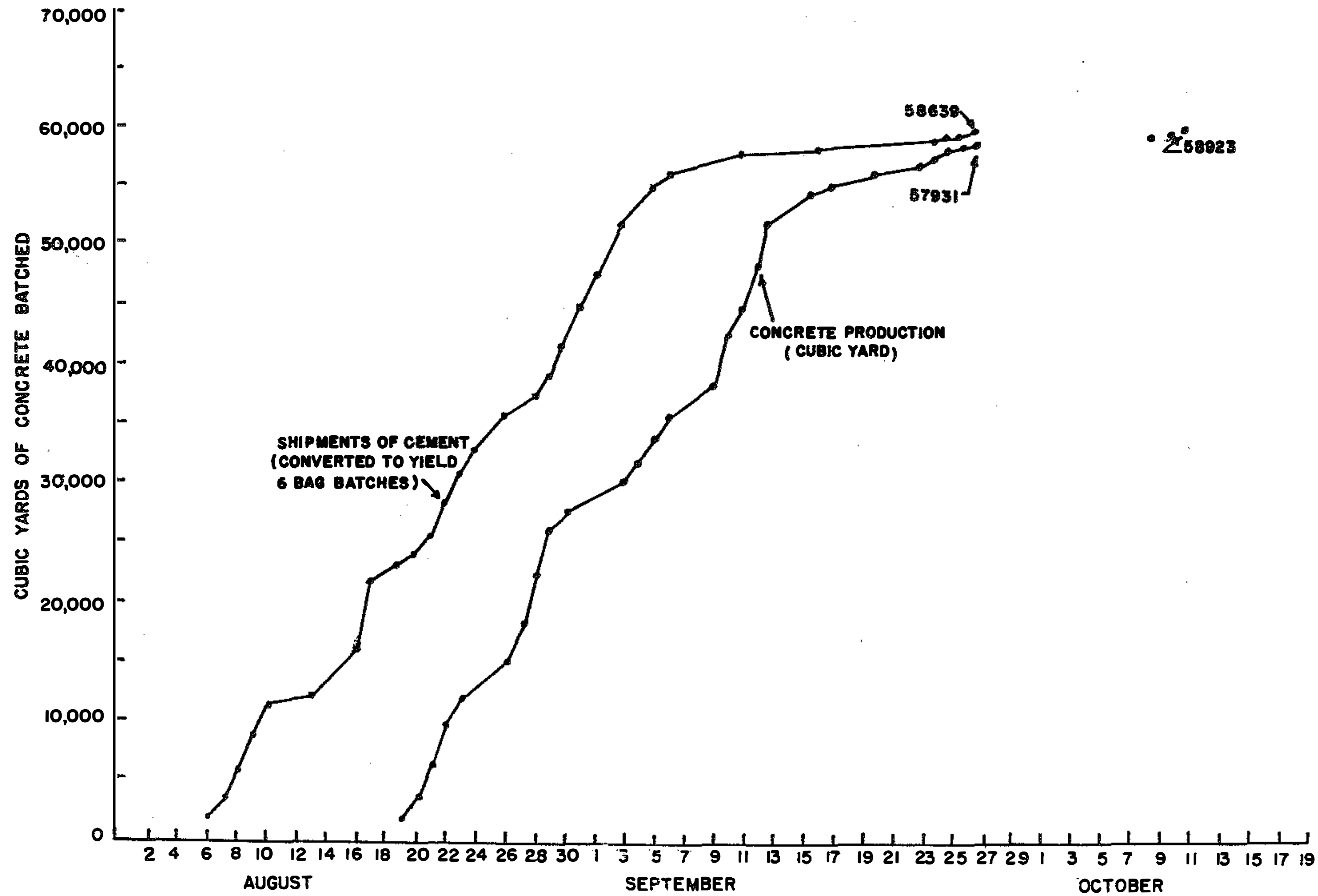


Figure 1 Accumulative Graph, Showing Cement Shipments and Volume of Concrete Manufactured.

TABLE I: Core Data - First Twenty are Check Cores; Forty-five Additional Specimens are from Third Coring

CORE NUMBER	STATION	LANE	COMPRESSIVE STRENGTH (psi)	SSD UNIT WEIGHT (pcf)	OD UNIT WEIGHT (pcf)	METHOD 1 PERCENT VOIDS	METHOD 1 THEORECTICAL SOLID UNIT WEIGHT (pcf)	METHOD 2 PERCENT VOIDS	METHOD 3 PERCENT VOIDS	METHOD 3 THEORECTICAL SOLID UNIT WEIGHT (pcf)
133B(A)	2312 + 10	Rt,EB	2885	140.8	134.5	18.389	164.83	17.991	17.769	163.63
29 (B)	2445 + 03	Lt,EB	2600	141.8	134.8	17.753	163.99	17.808	18.191	164.80
64 (C)	2428 + 67	Lt,WB	3030	141.8	134.6	17.753	163.75	17.931	18.512	165.15
10 (D)	2249 + 70	Lt,EB	3610	142.3	135.7	17.481	164.49	17.260	17.250	164.09
59 (E)	2373 + 70	Lt,EB	3250	141.5	134.0	17.935	163.22	18.296	19.192	165.84
133 (F)	2312 + 10	Rt,EB	2885	141.6	135.1	17.935	164.56	17.625	17.590	163.96
64B(G)	2428 + 67	Lt,WB	3030	141.8	135.2	17.753	164.48	17.564	17.550	164.08
26 (H)	2415 + 46	Lt,EB	2780	141.8	134.7	17.753	163.87	17.869	18.351	165.08
137 (I)	2352 + 15	Rt,EB	3175	140.2	133.5	17.752	162.41	18.601	17.810	162.41
58 (J)	2363 + 73	Lt,WB	3160	142.4	135.2	17.390	163.68	17.564	18.111	165.08
139 (K)	2372 + 21	Rt,EB	3160	145.1	138.1	15.756	164.01	15.796	15.991	164.41
97 (L)	2318 + 03	Rt,WB	3900	143.5	136.7	16.754	164.30	16.650	16.771	164.11
16 (M)	2309 + 91	Lt,EB	2960	139.8	132.9	19.024	164.07	18.967	19.431	164.89
27 (N)	2425 + 27	Lt,EB	3140	144.4	137.9	16.210	164.56	15.918	15.690	163.58
139B(O)	2372 + 21	Rt,EB	3610	142.8	135.8	17.208	164.01	17.199	17.591	164.81
143 (P)	2417 + 63	Rt,EB	3500	144.2	137.3	16.300	164.04	16.284	16.431	164.23
147 (Q)	2464 + 20	Rt,EB	3210	143.7	136.9	16.664	164.35	16.528	16.671	164.35
134 (R)	2321 + 91	Rt,EB	2885	141.7	135.3	17.844	164.60	17.504	17.329	163.60
8 (S)	2229 + 57	Lt,EB	3175	141.0	134.5	18.300	164.63	17.991	17.990	164.02
107 (T)	2423 + 07	Rt,WB	3140	142.5	135.4	17.390	163.92	17.443	17.951	165.12
1	2190 + 15	O,EB	4454	144.4	(137.59)	16.210	164.18	16.110	16.195	164.18
1A	2190 + 15	O,EB		143.3						
1AT	2190 + 15	O,EB	3947	143.0	(136.25)	17.027	164.16	16.924	16.989	164.16
1AB	2190 + 15	O,EB	4938	143.0	(136.25)	17.027	164.16	16.924	16.989	164.16
2	2190 + 15	Cl,EB	4174	142.5	(135.78)	17.390	164.38	17.214	17.350	164.38
2A	2190 + 15	Cl,EB		143.0						
2AT	2190 + 15	Cl,EB	3023*	143.0	135.7	17.027	163.50	17.260	17.872	165.29
2AB	2190 + 15	Cl,EB	3891*	142.7	138.7	17.027	167.11	15.430	12.583	158.70
3	2190 + 15	M,EB	2926*	144.0	140.2	16.391	167.70	14.516	11.563	158.37
3A	2190 + 15	M,EB		143.1						
3AT	2190 + 15	M,EB	3301*	140.0	137.8	17.662	167.44	15.979	13.604	159.50
3AB	2190 + 15	M,EB	3348*	143.9	140.3	16.482	168.02	14.455	11.342	158.17
4	2342 + 61	O,EB	3508	142.6	(135.87)	17.299	164.29	17.156	17.258	164.29
4A	2342 + 61	O,EB		140.6						
4AT	2342 + 61	O,EB	2840	139.2	(132.63)	19.387	164.55	19.132	19.302	164.35
4AB	2342 + 61	O,EB	3708	142.0	(135.30)	17.662	164.40	17.505	17.613	164.20
5	2342 + 61	Cl,EB	3430	141.4	(135.01)	18.025	164.65	17.679	17.510	163.65
5A	2342 + 61	Cl,EB		140.8						
5AT	2342 + 61	Cl,EB	2736*	138.6	133.7	19.750	166.71	18.479	17.026	161.08
5AB	2342 + 61	Cl,EB	3328*	142.8	138.8	17.208	167.63	15.369	12.783	159.17
6	2342 + 61	M,EB	2611*	140.9	136.5	18.389	167.28	16.772	14.724	160.02
6A	2342 + 61	M,EB		141.6						
6AT	2342 + 61	M,EB	2675*	140.2	132.7	18.752	163.42	19.089	20.092	166.08
6AB	2342 + 61	M,EB	3822*	142.7	135.8	17.201	164.01	17.200	17.431	164.41
7	2352 + 15	O,EB	3974	141.6	(134.92)	17.935	164.33	17.737	17.883	164.33
7A	2352 + 15	O,EB		141.5						
7AT	2352 + 15	O,EB	4031	141.5	(134.82)	17.935	164.22	17.795	17.875	164.22
7AB	2352 + 15	O,EB	3836	141.5	(134.82)	17.935	164.22	17.795	17.875	164.22
8	2352 + 15	Cl,EB	3360	141.7	(135.01)	17.844	164.25	17.679	17.791	164.25
8A	2352 + 15	Cl,EB		140.5						
8AT	2352 + 15	Cl,EB	2363*	140.6	136.0	18.570	167.08	17.077	15.245	160.38
8AB	2352 + 15	Cl,EB	2891*	140.5	136.4	18.570	167.57	16.833	14.444	159.35
9	2352 + 15	M,EB	2692*	142.4	137.5	17.390	166.47	16.162	14.426	160.63
9A	2352 + 15	M,EB		142.1						
9AT	2352 + 15	M,EB	3078*	141.1	136.1	18.207	166.38	17.016	15.486	161.07
9AB	2352 + 15	M,EB	3474*	142.6	135.2	17.299	163.48	17.564	18.332	165.48
10	2362 + 24	O,EB	3579	139.6	(133.01)	19.115	164.42	18.899	19.031	164.21
10A	2362 + 24	O,EB		139.5						
10AT	2362 + 24	O,EB	3641	139.7	(133.11)	19.115	164.94	18.841	19.039	164.33
10AB	2362 + 24	O,EB	3809	139.4	(132.82)	19.297	164.59	19.015	19.216	164.38
11	2362 + 24	Cl,EB	4343	146.0	(139.11)	15.211	164.05	15.180	15.215	164.05
11A	2362 + 24	Cl,EB		146.1						
11AT	2362 + 24	Cl,EB	3115*	142.0	134.3	17.662	163.18	18.113	19.213	166.21
11AB	2362 + 24	Cl,EB	5397*	149.9	145.3	12.760	166.28	11.406	8.845	159.32
12	2362 + 24	M,EB	2450*	142.3	137.8	17.481	167.03	15.979	13.885	160.05
12A	2362 + 24	M,EB		142.7						
12AT	2362 + 24	M,EB	2967*	142.0	133.7	17.662	162.45	18.479	20.174	167.54
12AB	2362 + 24	M,EB	5260*	152.1	148.1	11.398	167.16	9.699	17.783	173.83
13	2507 + 36	O,EB	2853	137.8	(131.30)	20.295	164.74	19.944	20.195	164.53
13A	2507 + 36	O,EB		138.3						
13AT	2507 + 36	O,EB	3030	139.0	(132.44)	19.569	164.73	19.247	19.486	164.52
13AB	2507 + 36	O,EB	2960	137.4	(130.92)	20.568	164.88	20.177	20.466	164.67
14	2507 + 36	Cl,EB	2582	138.7	(132.15)	19.750	164.78	19.422	19.663	164.58
14A	2507 + 36	Cl,EB		140.4						
14AT	2507 + 36	Cl,EB	2724*	141.5	136.3	17.935	166.02	16.894	15.506	161.30
14AB	2507 + 36	Cl,EB	2585*	139.1	133.7	19.478	166.09	18.479	17.527	162.06
15	2507 + 36	M,EB	2319*	140.8	135.9	18.389	166.54	17.138	15.526	160.83
15A	2507 + 36	M,EB		142.0						
15AT	2507 + 36	M,EB	2836*	142.3	135.1	17.481	163.76	17.625	18.212	165.16
15AB	2507 + 36	M,EB	2744*	141.2	133.9	18.207	163.70	18.357	19.172	165.72

*Specimens tested for strength after they were oven-dried and re-saturated.

O - 1.5 feet from outside edge of pavement

Cl - 2 feet from centerline

M - 1.5 feet from median edge of pavement

T - Top half of full core

B - Bottom half of full core

OD Wts. in () were synthesized: $\frac{\text{SSD Wt}}{145.948} \times 139.06 = \text{OD Unit Wt.}$

These values were used in Methods 2 and 3 when only the SSD Unit Weights were given.

Percentages of Voids which are underlined were selected for plotting against Compressive Strength.

Method 1: $\%\ \text{Voids} = 6 + 100\left(1 - \frac{\text{SSD Unit Wt.}}{145.948}\right) + 9.211 \times \frac{\text{SSD Unit Wt.}}{145.948}$

Method 2: $\%\ \text{Voids} = 100\left(1 - \frac{\text{OD Unit Wt.}}{164.007}\right)$

Method 3: $\%\ \text{voids} = 100\left(\frac{\text{SSD Unit Wt.} - \text{OD Unit Wt.} - 1.14}{62.4}\right) + 6 + 100\left(1 - \frac{\text{SSD Unit Wt.}}{145.948}\right)$

Theoretical Solid Unit Wt. $= 100\left(\frac{\text{OD Unit Wt.}}{100 - \%\text{Voids}}\right)$

The "A" companion of the "O" cores was sawed and the tops and bottoms were tested as individual specimens. Only the SSD unit weight and strengths were determined. The "A" companion was cut in two; however, the SSD unit weight of the whole core was determined beforehand. The SSD unit weight and the OD unit weight were determined on the top and bottom specimens individually. The strengths were determined after oven drying and then soaking (these strengths are marked with an asterisk). In the "M" cores, the first was treated as a whole specimen; both SSD unit weight and OD unit weight were determined; and, of course, the strength is marked with an asterisk. The "A" companion was soaked in order to obtain the SSD unit weight and then was sawed in two. Both SSD and OD unit weights were determined; strengths are marked with an asterisk.

Preliminary Analysis of Data

Core strengths and unit weight data were plotted as shown in Figs. 2 and 3. A trend-like relationship there seems to indicate that the heavier cores gave the higher strengths. There is, however, extensive scatter. For this reason, more discrete analysis of unit weights and voids seemed appropriate.

Voids-vs-Strength Hypothesis

If concrete contains excessive voids, its strength will be low. Conceptually, voids occur in the mortar phase. Generally speaking, if the voids are uniformly distributed -- as in the case of entrained air bubbles -- air trends to fluidize the fresh concrete in the same way that extra mixing water would. Therefore, when air is included, a saving of water should be realized; this, of course, reduces the water-cement ratio and strengthens the mortar commensurately. Thus, within limits, no loss of strength need occur. It is possible to effect an increase in strength with air entrainment -- up to 5 or 6% air; but, unless full advantage is taken of the opportunity to withhold mixing water, strength will decay as the air (voids) is increased.

In the case of hardened concrete, an estimate (statistical average) of the void volume may be made by microscopically examining sawed and polished surfaces (linear traverse method). The voids measured in this way include only discrete bubbles and large spaces. Conceptually, the air in these spaces was compressible when the concrete was freshly mixed. Additional voids occur because of excess (evaporable) mixing water; they are not detectable by ordinary microscopic examination. The total or combined voidage affects the strength of the concrete.

The design formula and batching proportions employed on the project provided the basis for the information given in Table II.

In hardened, dry concrete, the total void volume may be defined as:

$$\%\ \text{Total Voids} = 100 - \%\ \text{Solids}$$

The percentage of solids may be calculated from the OD unit weight and the apparent specific gravity of the solids -- that is:

$$\%\ \text{Solids} = 100 \times \frac{\text{OD Unit Weight}}{\text{Apparent Specific Gravity of Solids}}$$

Note: *This apparent specific gravity is the apparent gravity of the mortar solids combined with the OD bulk specific gravity of the coarse aggregate; when multiplied by 62.4, it yields the theoretical solid unit weight (free of air voids in the mortar and free of all evaporable water).*

Alternatively, the above equation may be written as:

$$\%\ \text{Solids} = 100 \times \frac{\text{OD Unit Weight}}{\text{Theorectical Solid OD Unit Weight}}$$

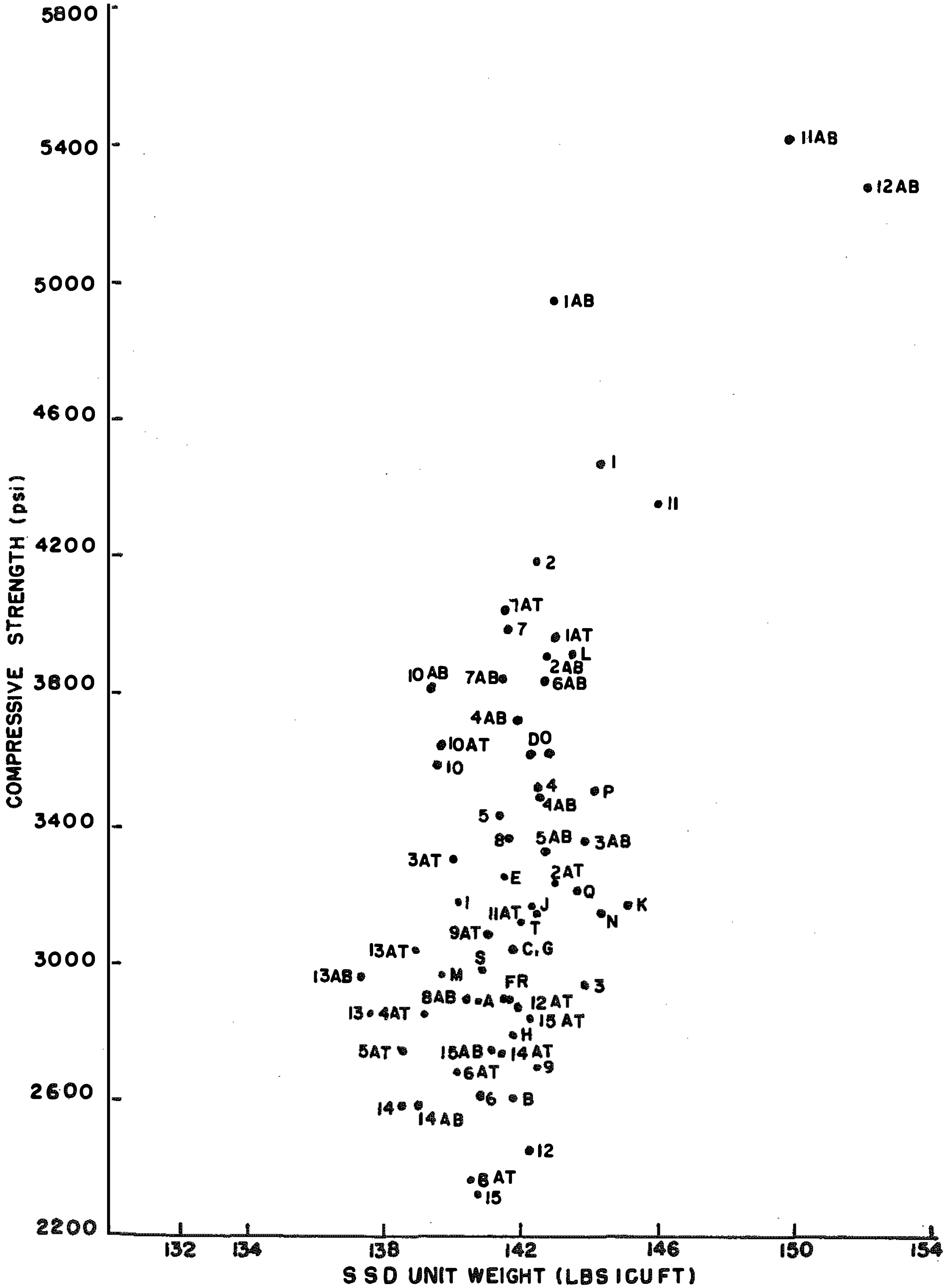


Figure 2 SSD Unit Weights versus Compressive Strengths.

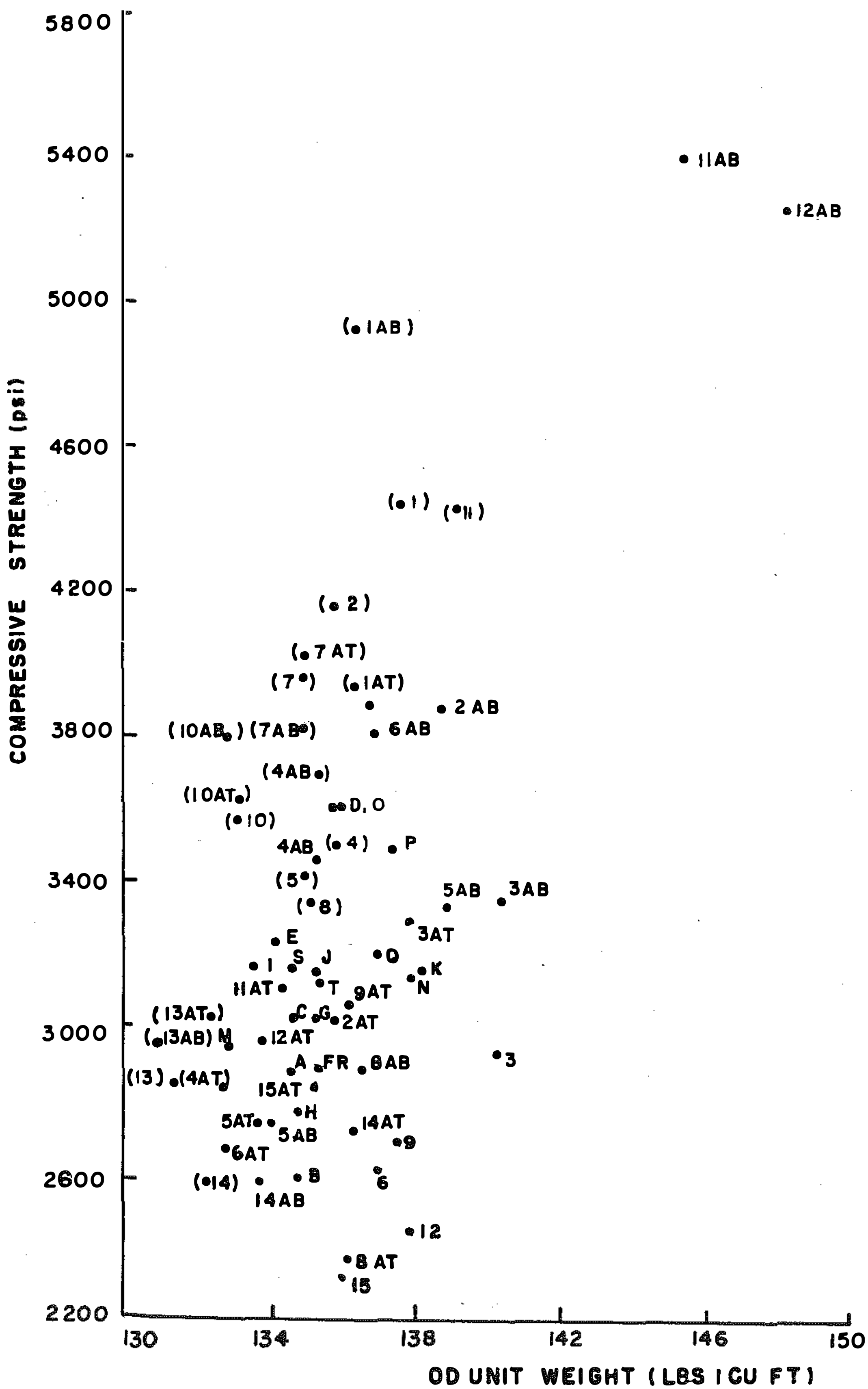


Figure 3 OD Unit Weights versus Compressive Strengths.

TABLE II ANALYSIS OF MIX-DESIGN FORMULA

Design Unit Weight (air-free basis)	- 153.7 lb/cuft
Design Unit Weight (for 6% air)	- 144.5 lb/cuft
Net Mixing Water (for 6% air)	- 9.4 lbs
Water Required for Hydration	- 5.1 lbs
SSD Unit Weight of Aggregate and Cement	- 135.1 lb/ cuft
Weight of Dry Cement (94 x 6/27)	- 20.9 lbs
SSD Weight of Aggregates	- 114.2 lbs
Evaporable Water in Aggregates (1% Estimate)	- 1.14 lbs
Total Evaporable Water (9.4 - 5.1 + 1.14)	- 5.44 lbs
Theoretical Dry Unit Weight of Cured Concrete (144.5 - 5.44)	- 139.06 lb/cuft
Summation of Voids in Mortar:	
Voids Due to Evaporable Water (4.3/62.4) x 100	- 6.891%
Voids Due to Densification of Hydration Water (1.00 - 0.7161) x 5.1/62.4	- 2.320%
Total Voids Attributed to Mixing Water	- 9.211%
Voids Due to Entrained Air	- 6.000%
Total Theoretical Voids; Dry, Cured Concrete	- 15.211%
Theoretical, Maximum Dry Unit Weight of Solids [139.06 / (1.000 - 0.1521)]	- 164.007 lb/cuft
Theoretical Apparent Specific Gravity of Solids (164.007 / 62.4)	- 2.628

Similar logic may be applied to fresh concrete to estimate air contents, i.e.

$$\% \text{ Air}^* = 100 - 100 \times \frac{\text{Measured Unit Weight of Fresh Concrete}}{\text{Theorectical or Design Unit Weight (air-free basis)}}$$

**Entrained + entrapped voids*

If it may be reasonably assumed that re-saturation (soaking) of a specimen of hardened concrete restores or reconstitutes the water content to approximately that of the fresh concrete, then:

$$\% \text{ Air} = 100 - 100 \times \frac{\text{SSD Unit Weight of Concrete Specimen}}{\text{Theorectical or Design Unit Weight (air-free basis)}}$$

Considering the volume of water re-absorbed,

$$\% \text{ Voids}^* = \frac{\text{SSD Unit Weight} - \text{OD Unit Weight}}{62.4}$$

**Attributable to loss of uncombined water*

Combining the two types of voids (above) gives a crude estimate of the total voidage in a specimen of hardened concrete. Taking Core 15 as an example:

$$100 \times (1 - 140.8/153.7) = 8.40\%$$
$$100 \times (140.8 - 135.9/62.4) = 7.85\%$$
$$\text{Total Voids} = 16.25\%$$

Theoretical Solid Unit Weight = 135.9/(1.0000 - 0.1625) = 162.3 lbs/cu ft

Apparent Specific Gravity of Solids = 162.3/62.4 = 2.60

If the above rationale is correct and applicable, the theoretical solid unit weights -- and, of course, the apparent specific gravities of the solids -- would be a constant for all specimens of concrete taken from the pavement. Unfortunately, 162.3 (for Core 15) does not compare favorably with 164.0 as obtained from the design formula. Alternative methods provide better values in this particular case.

For example: 8.40% + 9.21% (from design formula) = 17.61% voids; using the OD unit weight of the specimen: 135.9/(1 - 0.1761) = 164.94, which is closer to the theoretical value. Using only the OD unit weight and the theoretical solid unit weight of cured, dry concrete (from design formula) yields: (1 - 135.9/164.0) x 100 = 17.14% voids.

Ideally, the several methods would yield precisely the same results.

Constancy of the theoretical solid unit weight provided a criterion for selecting the most rational value of voids and also confirmed the hypothesis stated previously (that the theoretical solid unit weight should be constant for all specimens if the batch proportions were consistent). All remaining variances (errors) are then attributable to inaccuracies in measurements (weights).

Summarily, three methods were employed. When only the SSD unit weight was known, only Method 1 was directly applicable. The methods are defined as follows:

Method 1

$$\% \text{ Voids} = 100\left[1 - \frac{\text{SSD Unit Weight}}{145.948}\right] + 9.211 \times \frac{\text{SSD Unit Weight}}{145.948}$$

Method 2 $$\% \text{ Voids} = 100 \left[1 - \frac{\text{OD Unit Weight*}}{164.007}\right]$$

$$\text{* When only SSD Unit Weight is given, use } \frac{\text{SSD Unit Weight}}{145.948} \times 139.06$$

Method 3

$$\% \text{ Voids} = 100 \left[\frac{\text{SSD Unit Weight} - \text{OD Unit Weight} - 1.14}{62.4}\right] + 6 + 100\left[-\frac{\text{SSD Unit Weight}}{145.948}\right]$$

The results obtained by the respective methods are given in Table I. TheOD unit weight was not used directly in Method I; it was possible, therefore, to use the percentages of voids calculated by Method I and the OD unit weight (when given and/or synthesized) to resolve a theoretical solid unit weight as a validity check upon the percentage of voids and also on the constancy hypothesis. These values appear to the right of Method 1 in Table I. Method 2 presumes constancy of the theoretical solid unit weight; if this premise is valid, the only possible error is in the OD unit weight of the specimen. Agreement between Methods 1 and 2 tends to validate all values. Method 3 utilizes two measured parameters and one mix-design parameter; any errors in obtaining re-saturation and a realistic SSD unit weight or OD unit weight are somewhat compounded in this method; consequently the percentages of voids obtained are most likely to be lower -- this would yield low values for the theoretical solid unit weight also. Methods 1 and 3 contain a correction (estimated from the design formula) to reflect the densification which the water of hydration underwent as the concrete hardened; in other words, the specimens conceivably could have regained more water than the fresh concrete contained originally. According to the design-formula analysis, the densification would permit admittance of 2.32% (0.0232 cu ft per cu ft or 1.448 lbs per cu ft) more water than the fresh concrete contained. The design unit weight of fresh concrete containing 6% air was 144.5 pcf; adding 1.448 yields 145.948pcf. Method 3 contains an additional correction which, in effect, restores the "absorbed" water to the aggregates in the OD unit weight -- or subtracts it from the SSD unit weight. The difference between SSD unit weight and OD unit weight, when adjusted in this way, reflects the weight on water absorbed by the mortar only -- and this weight of water divided by 62.4 yields a volume equal to the volume of voids in the mortar portion of the dry concrete. To be completely precise, these two correction factors should vary slightly in proportion to unit weights; however, in these calculations, they were considered to be constants.

The most rational percentages of voids were selected from Table I (underlined) and plotted against their respective strengths. The resulting relationship is shown in Fig. 4. The correlation appears to be quite good, and the voids-strength hypothesis stands confirmed.

An interesting observation is that at 15.211% voids (design value), the strength interpolated from the curve would be about 4800 psi. Unfortunately, the majority of specimens exceeded this percentage of voids and therefore suffered a corresponding loss of strength. It may be noted also that 2% excess air (2 over 6) or a total of 17.211% would barely have permitted the concrete to achieve 4000 psi.

The excess voids may be attributed to the following possible causes:

1. Excess entrained air (and records indicate this to be a partial cause).
2. Gasing.
3. Excess mixing water.
4. Inadequate consolidation of the concrete *in situ* (visual and microscopic examination of the cores indicate this to be at least a partial cause).

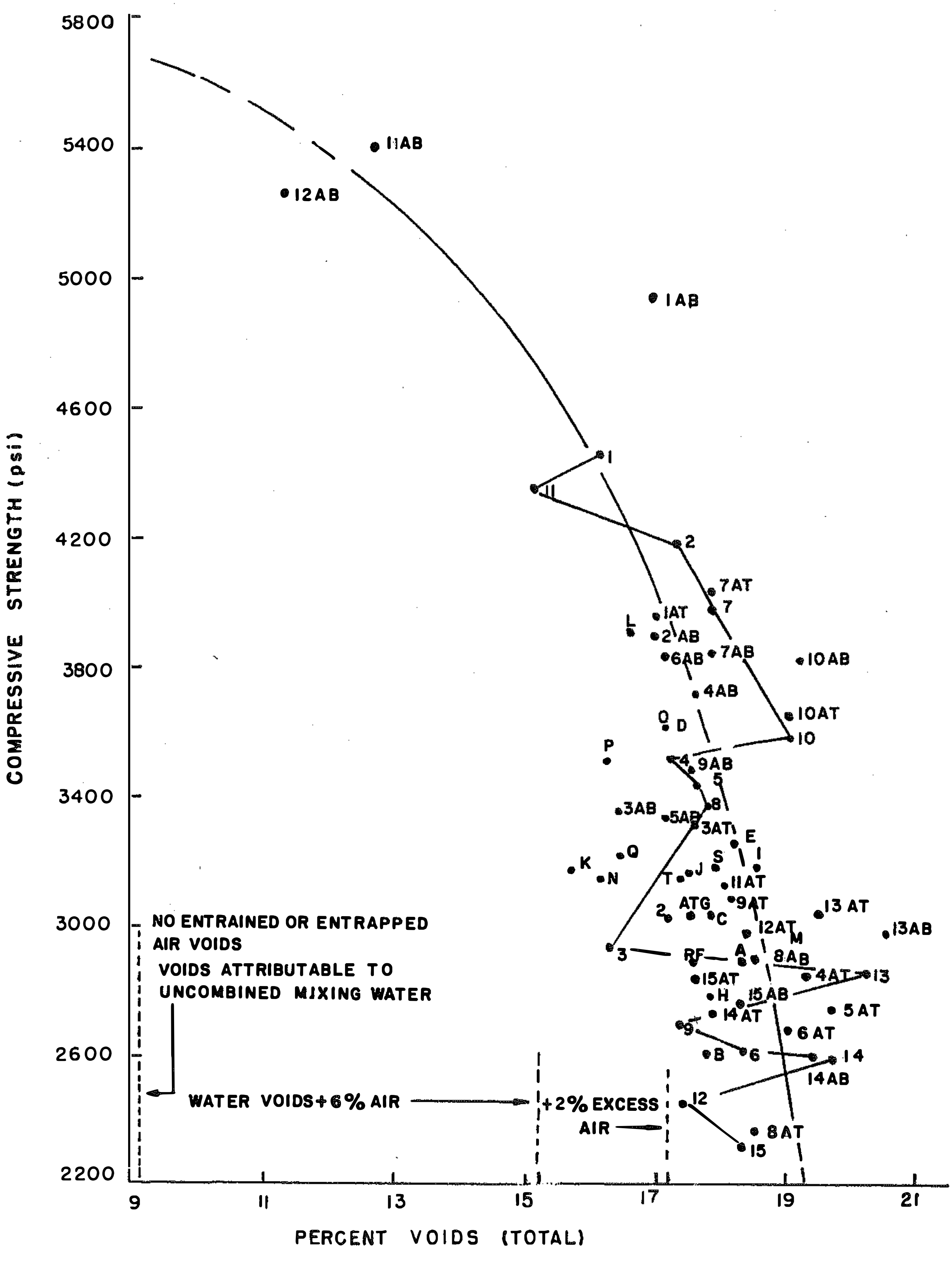


Figure 4 Relationship between Compressive Strength and Total Voids.

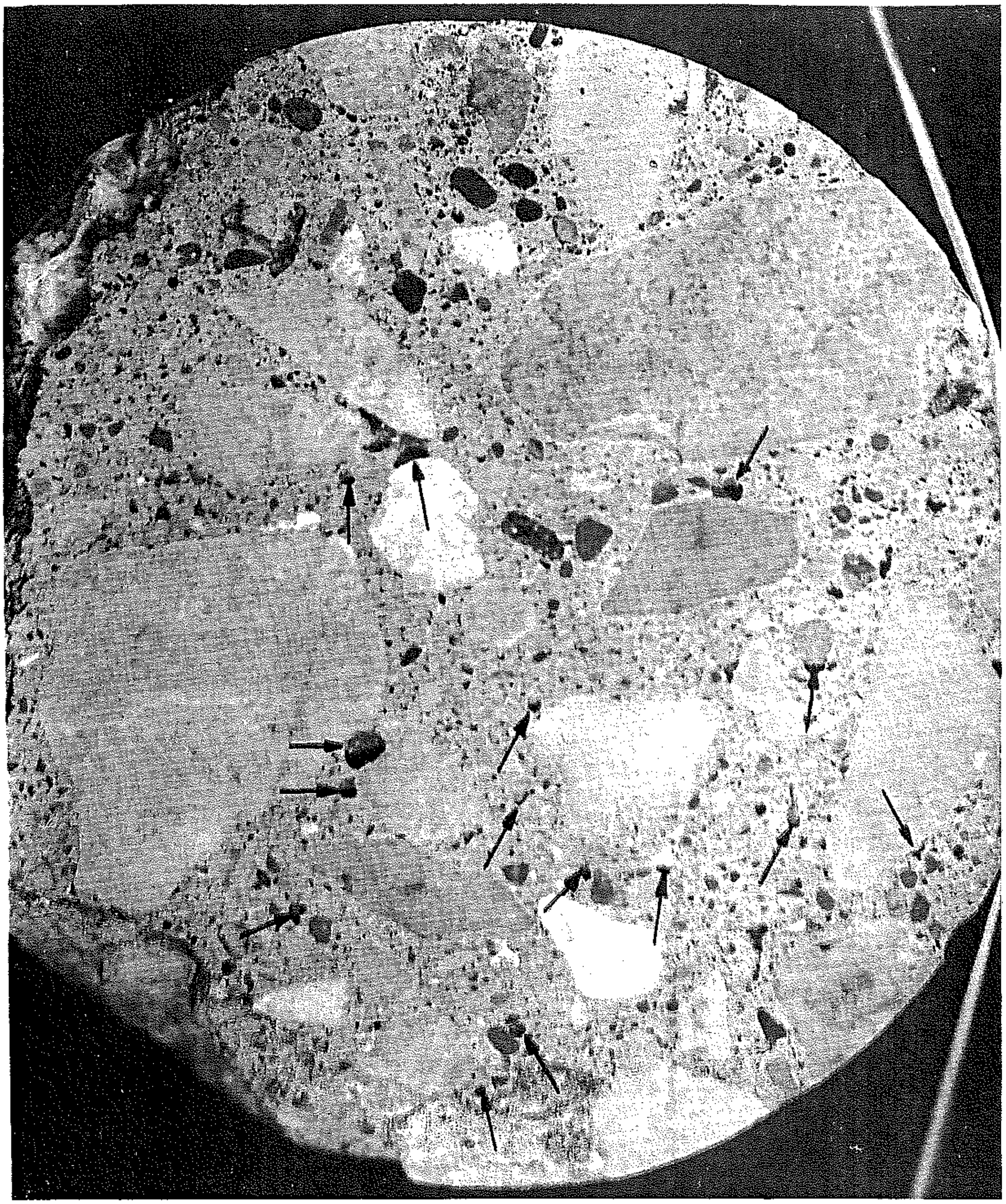


Figure 5 Photomacrograph of Cross Section through Core 15. Arrows indicate voids.

Microscopic Measurements of Voids

Several core remnants were sawed and polished for microscopic measurement of voids (linear traverse method, ASTM C457). The designed air content, of course, was 6%. The pressure meter measurements on the fresh concrete generally exceeded 6% but did not exceed 8%. Microscopically-determined air voids which significantly exceed 8% naturally reflect inadequate consolidation of the concrete ***in situ***. Over-dosing of air-entraining agent would presumably have been detected by air measurements on the job.

A photomacrograph of a typical cross-section through Core 15 (approximately 1 inch from bottom) is shown in Fig. 5. The types of voids measured by linear traverse are shown by arrows.

Note: Air voids determined microscopically should be considerably less than the total voidage as calculated from the OD unit weight and theoretical solid unit weight.

The percentages of voids obtained by this method (on selected cores) were adjusted to a total-voids basis and then superimposed on the graph shown previously as Fig. 4. Fig. 6 shows the superimposed values and the basis for adjusting the discrete-void percentages to a total-voids basis. The agreement between these values and those obtained from SSD and OD unit weights is remarkable indeed. Where discrepancies occur, the possibility remains that the slice taken through the core was not truly representative of the core. This appears to be so in the case of Core 11 AT. The slice was taken near the bottom of the specimen; the slice taken for 11 AB was near the top of the specimen; since these specimens were actually the top and bottom half of a core, both slices measured by linear traverse were near the mid-height of the original core. The top half had a low strength and low unit weights. Consequently, the upper reaches of 11 AT must have been very porous.

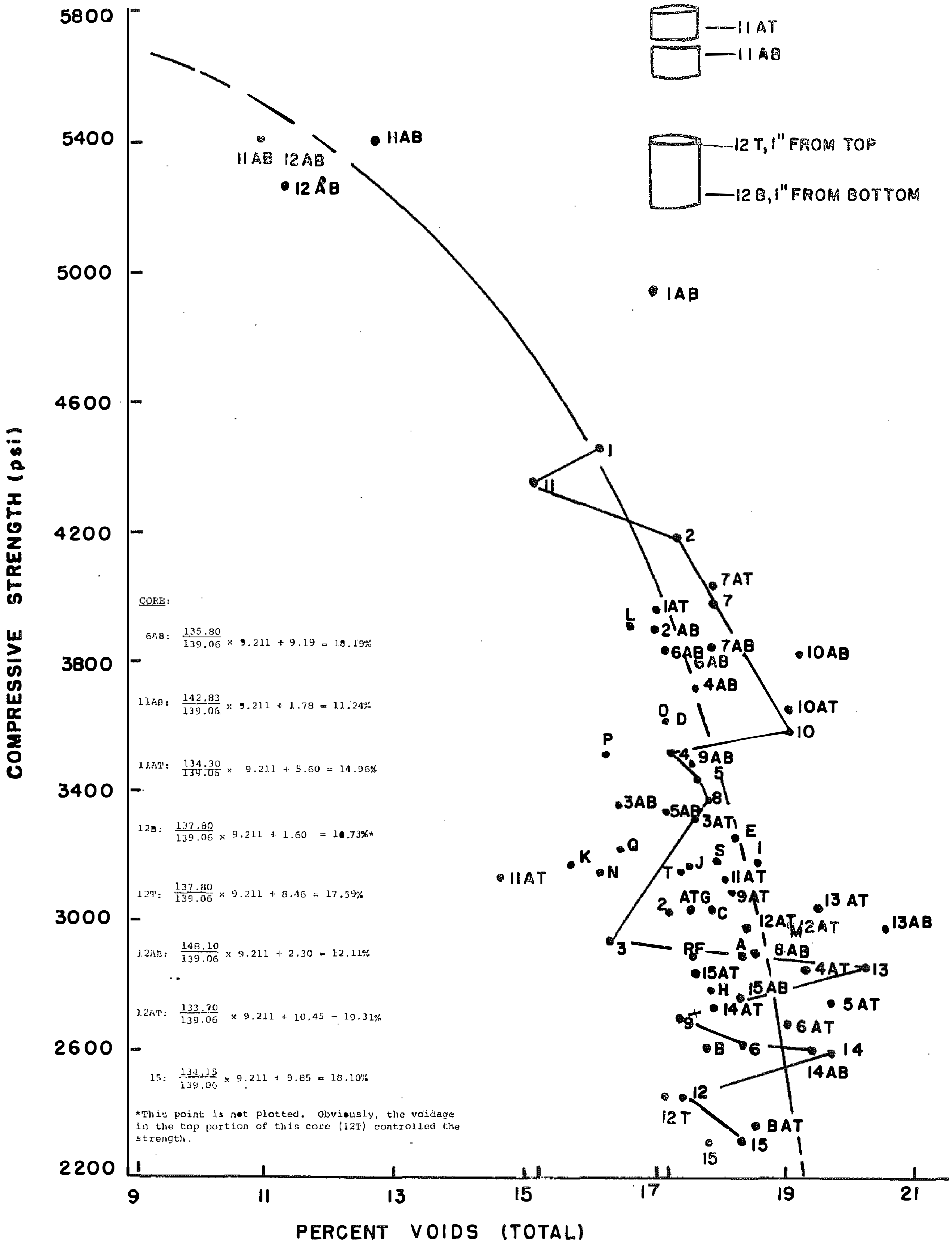


Figure 6 Voidages Determined by Linear Traverse Converted to Total Voids; Superimposed on Figure 4.

APPENDIX I

Cylinder and Beam Strength Data

CYLINDERS

CYLINDER NO	DATE MADE	DATE TESTED	STATION A	COMPRESSIVE STRENGTH (psi) A	STATION B	COMPRESSIVE STRENGTH (psi) B	STATION C	COMPRESSIVE STRENGTH (psi) C	REMARKS
1-AB	8-19-68	9-16-68	2526+50	4170	2515+50	3460	------EBL	----	
2-ABC	8-20-68	9-17-68	2507+50	3005	2494+00	4065	2488+00	3785	
3-ABC	8-21-68	9-18-68	2473+15	4065	2460+25	3325	2450+10	3995	
4-ABC	8-22-68	9-19-68	2439+25	3290	2418+25	3680	2418+25	4315	
5-ABC	8-23-68	9-20-68	2385+25	3890	2365+75	4125	2365+75	3785	
6-ABC	8-26-68	9-23-68	2358+75	3145	2339+00	3675	2331+00	3075	
7-ABC	8-27-68	9-25-68	2300+80	4105	2295+00	4140	2286+10	4530	
8-ABC	8-28-68	9-26-68	2282+75	3890	2250+30	4115	2242+40	4115	
9-ABC	8-29-68	9-27-68	2227+40	4420	2185+60	4670	2185+60	4530	
10-ABC	8-30-68	9-27-68	2175+10	4315	2158+70	3995	2156+40	4350	
11-AB	9-03-68	10-1-68	2525+75	3820	2508+50	4175	-----WBL	----	
12-ABC	9-04-68	10-4-68	2486+00	4490	2477+75	4420	2470+00	4420	
13-AB	9-05-68	10-3-68	2468+25	4105	2451+60	4490	-----	----	
14-ABC	9-06-68	10-4-68	2450+75	4280	2435+30	4070	2425+25	4775	
15-ABC	9-09-68	10-7-68	2419+80	3785	2408+00	4145	2383+50	4350	
16-ABC	9-10-68	10-8-68	2379+50	4455	2347+20	4985	2339+15	5095	
17-ABC	9-11-68	10-9-68	2325+80	4970	2298+50	4880	2286+90	4880	
18-ABC	9-12-68	10-10-68	2281+35	4705	2258+50	4880	2250+50	4950	
19-ABC	9-13-68	10-11-68	2232+25	4670	2207+15	5215	2201+75	4670	
20-ABC	9-16-68	10-14-68	2184+60	3930	2176+10	4145	2163+40	3640	
21-A	9-17-68	10-15-68	2162+00	4490	-----	----	-----	----	
22-A	9-18-68	10-16-68	2530+10	4455	-----	----	-----	----	
23-AB	9-19-68	10-17-68	2314+10	4950	2445+65	5055	-----	----	
24-ABC	9-20-68	10-18-68	2530+00	4810	2444+90	3890	2396+00	3890	
25-ABC	9-23-68	10-21-68	17+80 D	4140	13+50 D	4105	5+80 D	5095	(Ramps)
26-ABC	9-24-68	10-22-68	2530+35	3255	0+25 C	3040	3+60 C	4490	(ML Sta = EBL)
27-ABC	9-25-68	10-23-68	16+00 C	5090	2400+95	4950	2396+00	5020	(ML Sta = EBL)
28-ABC	9-26-68	10-24-68	14+75 C	4380	2529+90	4550	2313+90	4770	(ML Sta = EBL)
29-AB	9-27-68	10-25-68	2483+70	4920	2517+35	4350	-----EBL	----	

Rowan I-64-6(15)130
PCC Pavement

BEAMS

BEAM NO	DATE MADE	DATE TESTED	FLEXURAL STRENGTH (psi)	STATION	REMARKS
1-A	8-19-68	8-29-68	583	2521+50 EBL	Passed
2-A	8-20-68	8-30-68	633	2501+70	Passed
2-B	8-20-68	8-30-68	508	2501+70	Failed (2-B was inadvertedly broken first)
3-A	8-21-68	8-29-68	558	2472+05	Passed
4-A	8-22-68	8-29-68	483	2430+00	Failed
4-B	8-22-68	9-03-68	533	2430+00	Failed
5-A	8-23-68	8-30-68	475	2372+70	Failed
5-B	8-23-68	9-03-68	483	2372+70	Failed
6-A	8-26-68	9-03-68	550	2352+00	Passed
7-A	8-27-68	9-03-68	566	2311+10	Passed
8-A	8-28-68	9-05-68	591	2259+75	Passed
9-A	8-29-68	9-05-68	591	2201+00	Passed
10-A	8-30-68	9-05-68	608	2158+70	Passed
11-A	9-03-68	9-11-68	575	2525+75 WBL	Passed
12-A	9-04-68	9-11-68	550	2491+90	Passed
13-A	9-05-68	9-11-68	700	2457+30	Passed
14-A	9-06-68	9-13-68	308	2443+25	Failed
14-B	9-06-68	9-17-68	633	2443+25	Passed
15-A	9-09-68	9-16-68	533	2414+40	Failed
15-B	9-09-68	9-19-68	700	2414+40	Passed
16-A	9-10-68	9-17-68	700	2359+60	Passed
17-A	9-11-68	9-19-68	666	2317+30	Passed
18-A	9-12-68	9-19-68	650	2270+90	Passed
19-A	9-13-68	9-20-68	733	2221+00	Passed
20-A	9-16-68	9-24-68	625	2192+60	Passed
21-A	9-17-68	9-24-68	716	2159+50	Passed
22-A	9-18-68	9-25-68	633	2478+90	Passed (Block)
23-A	9-19-68	9-26-68	600	2396+00	Passed
24-A	9-20-68	9-30-68	650	2484+20	Passed
25-A	9-23-68	9-30-68	591	17+18 Ra. D	Passed
26-A	9-24-68	10-2-68	525	2530+35 EBL	Failed
26-B	9-24-68	10-2-68	583	2530+35	Passed
27-A	9-25-68	10-2-68	616	16+00 Ra. C	Passed
28-A	9-26-68	10-3-68	666	2516+20 EBL	Passed (Wedge)
29-A	9-27-68	10-4-68	666	2401+05 EBL	Passed

APPENDIX II

Slump and Air Measurements

DATE	LANE	BEGINNING		ENDING		TEMPERATURE		TIME	SLUMP (inches)	AIR CONTENT (percent)
		STATION	TIME	STATION	TIME	AM	PM			
8-20-68	EB	2512+95	7:25	2484+60	6:30	73	93	7:50	3 1/4	7.2
								9:00	3 1/4	8.0
								9:15	3	7.2
								10:45	1 3/4	8.0
								11:30	2 1/4	4.0
								1:15	2 1/4	7.6
								3:15	2	6.2
								4:45	2	6.2
8-21-68	EB	2478+25	7:30	2449+45	3:20	72	92	7:50	3	7.0
								9:00	2 1/2	6.8
								9:45	2 1/4	6.2
								2:15	3	7.4
								4:00	2 1/4	6.6
8-22-68	EB	2449+45	7:30	2401+95	7:00	72	93	7:55	2 1/4	4.2
								8:15	2 1/4	4.6
								9:30	2 1/2	7.4
								12:30	3	6.7
								6:05	2 1/4	6.2
8-23-68	EB	2395+45	7:15	2365+00	3:00	70	91	7:50	2 1/2	6.4
								10:00	2 1/2	6.7
								12:00	3	6.4
								3:00	1 1/2	6.6
8-26-68	EB	2365+00	8:30	2323+30	6:00	59	79	8:45	2 1/2	6.7
								10:15	2 1/2	7.2
								12:00	2 1/2	6.8
								2:00	2	6.8
								4:00	2	6.6
8-27-68	EB	2323+30	7:15	2283+50	5:15	59	76	7:40	2 1/2	7.9
								11:15	2 1/2	7.4
								1:15	2 1/4	6.6
								2:45	2	6.6
								4:45	3	7.0
8-28-68	EB	2283+50	7:25	2229+65	7:15	57	77	8:00	2 1/2	7.6
								10:00	2	7.2
								11:45	2 1/2	7.4
								1:15	2 3/4	7.0
								5:00	2 1/4	6.4
8-29-68	EB	2229+65	8:00	2176+05	6:50	56	80	8:30	2 1/4	6.2
								10:00	2 3/4	7.2

								1:00	2 3/4	6.0
								3:30	1 3/4	5.6
								5:00	2 1/4	6.6
8-30-68	EB	2176+05	7:20	2156+00	1:30	54	82	8:00	2	5.6
								11:20	2 1/2	6.8
								12:15	2	5.5
9-3-68	WB	2529+50	8:20	2501+20	5:20	65	84	8:45	2 3/4	7.5
								10:30	2	6.8
								1:15	2	5.8
								3:30	3	6.4
9-4-68	WB	2501+20	7:15	2469+20	3:45	67	75	8:00	2	7.5
								10:45	2 1/2	6.4
								12:15	2 1/2	6.6
								1:30	2	5.8
								3:30	2 1/2	6.0
9-5-68	WB	2469+20	7:25	2451+50	Noon	68	80	8:00	2 3/4	7.0
								10:40	2 3/4	6.1
								12:35	1 1/2	5.7
9-6-68	WB	2451+50	7:15	2420+30	5:00	59	75	7:45	2 3/4	6.8
								11:30	2 3/4	6.8
								1:45	2 1/4	6.6
								3:40	2	6.9
9-9-68	WB	2420+30	8:15	2320+65	4:55	63	78	8:40	3	7.6
								10:00	2 1/4	6.5
								11:45	2 3/4	7.0
								3:00	2	7.4
								4:35	3	6.4
9-10-68	WB	2380+65	7:20	2327+50	6:30	58	67	7:45	3	6.8
								9:45	1 3/4	6.3
								11:35	2	5.3
								2:10	1 3/4	6.0
								4:15	2 1/2	6.6
9-11-68	WB	2327+50	7:30	2285+70	3:45	58	65	7:45	3	7.2
								9:30	2 1/4	6.1
								12:45	2 3/4	7.9
								1:15	2 3/4	7.3
								3:40	2 1/4	6.2
9-12-68	WB	2285+70	7:30	2242+65	5:40	51	74	8:00	2	6.9
								10:30	1 3/4	5.9
								12:15	2 1/2	6.3
								2:00	2 1/2	6.1
								2:45	2 1/2	7.3
								4:10	3	6.2

9-13-68	WB	2242+65	7:15	2198+20	4:30	48	77	7:45	3	7.5
								9:30	2 3/4	7.0
								11:45	1 1/2	5.3
								2:45	1 1/2	6.6
								4:00	1 1/2	5.6
9-16-68	WB	2198+20	8:20	2162+50	4:30	64	77	8:45	2 3/4	7.4
								10:10	2	5.8
								11:45	1 3/4	5.7
								1:45	2 3/4	6.6
								4:10	2	6.0
9-17-68	WB	2162+50	7:30	2155+95	9:50	63	78	7:50	2 3/4	5.4
								8:45	2 1/2	5.5

APPENDIX III

Tabulation of Original and Check Core Strengths

Project I 64-6(15)130

CORE NUMBER	STATION	LANE	ORIGINAL CORE STRENGTH (psi)	CHECK CORE STRENGTH (psi)
118	2159+54	Rt, EB	3175	3175
8	2229+57	Lt, EB	3465	3175
10	2249+70	Lt, EB	3390	3610
129	2271+19	Rt, EB	3465	3755
130	2281+39	Rt, EB	2960	3320
131	2292+03	Rt, EB	3320	3140
16	2309+91	Lt, EB	3390	2960
133	2312+10	Rt, EB	3250	2885
17	2320+00	Lt, EB	3175	2815
134	2321+91	Rt, EB	3320	2885
135	2332+02	Rt, EB	2885	2995
136	2342+61	Rt, EB	3250	2780
20	2350+09	Lt, EB	2885	2525
137	2352+15	Rt, EB	3105	3175
21	2360+30	Lt, EB	3030	3070
138	2362+24	Rt, EB	3250	2995
139	2372+21	Rt, EB	3250	3610
23	2380+12	Lt, EB	2885	3250
140	2382+24	Rt, EB	3250	3175
24	2390+00	Lt, EB	3250	2925
141	2391+94	Rt, EB	2885	2960
25	2405+00	Lt, EB	2600	2600
142	2406+27	Rt, EB	3175	3175
26	2415+46	Lt, EB	3250	2780
143	2417+63	Rt, EB	3250	3500
27	2425+27	Lt, EB	3105	3140
28	2435+25	Lt, EB	3105	2450
29	2445+03	Lt, EB	3030	2600
146	2450+18	Rt, EB	3030	2995
30	2455+09	Lt, EB	3175	3535
147	2464+20	Rt, EB	3030	3210
148	2472+24	Rt, EB	2960	2850
32	2474+94	Lt, EB	2740	2925
149	2486+06	Rt, EB	3465	2925
33	2490+00	Lt, EB	3105	2450
150	2497+39	Rt, EB	3105	3070
151	2507+36	Rt, EB	2165	2525
35	2510+00	Lt, EB	3250	2670
152	2516+72	Rt, EB	2815	3465
153	2526+36	Rt, EB	3410	3535
85	2198+00	Rt, WB	3250	2850
46	2242+60	Lt, WB	3105	3175
95	2297+92	Rt, WB	3320	3570
54	2322+27	Lt, WB	3465	3680
97	2318+03	Rt, WB	3320	3900
55	2332+28	Lt, WB	3175	3250
99	2339+06	Rt, WB	3465	3430
58	2363+73	Lt, WB	3465	3610
59	2373+70	Lt, WB	3320	3250
60	2383+13	Lt, WB	2815	3570
62	2407+70	Lt, WB	3320	3250

P age 2 (cont'd)

106	2413+00	Rt, WB	3250	3790
64	2428+67	Lt, WB	3105	3030
107	2423+07	Rt, WB	3390	3140
65	2438+98	Lt, WB	3250	3175
109	2443+15	Rt, WB	3320	3210
70	2495+36	Lt, WB	3465	3430
73	2526+42	Lt, WB	3465	3465
116	2517+27	Rt, WB	2885	2345
144	2427+85	Rt, EB	3465	3175

Note: Check Cores Tested 2-24-69

For Original Core Drill Report See:
Rowan Co. I 64-6(15)130
Lab No 32150 thru 32238
Date 12-17-68

Memorandum Report

PAVEMENT CORE STRENGTHS

[I 64 - (7)109, Bath-Montgomery Counties]

Division of Materials

October 13, 1969

MEMO TO: Dan C. Woodward
Division of Construction

J. H. Havens
Division of Research

FROM: John McChord, Director
Division of Materials

BY: W. H. Baker, Civil Engineer Pr. W. H. B
Concrete Section

DATE: October 13, 1969

SUBJECT: Cement Concrete Pavement
Core Strengths
Bath-Montgomery I-64-6-(7)-109

In a letter of July 17, Mr. Johnson of the Bureau of Public Roads requested that the subject project be included in a research investigation similar to that planned for the Rowan County I-64-6-(15) 130 project.

This project, as did the Rowan County project, had an abnormal number of core strengths testing below the 3500 psi level. However, there is in my opinion a significant difference between the two projects in that the average age of original mainline cores at time of test was <u>101</u> days for the Rowan County project as opposed to an average age of <u>38</u> days for the Bath-Montgomery project.

Also, a review of the pavement core drill report for this project (attached) reveals that a number of cores were tested at ages of 18 to 28 days. Further eview also shows that generally those tests below 3500 psi compressive strength coincide with cores tested within the above age range. (Forty of the total 49 mainline failures occurred within the 18 to 28 day range). This becomes more significant when it is noted that these are ages of test, not ages at which the actual coring was done. For instance, some of the cores tested at 18 days may have been actually drilled at a time considerably earlier than this (perhaps at 10 days of age) which would be at a time when cores might normally be expected to be more susceptible to damage through the process of drilling.

Due to the above, I am of the opinion that questions as regards the core strengths obtained on this particular project may be partially, if not entirely, satisfied by a close review of the age-strength relationships of cores already taken without the need for extensive additional taking of cores or other investigation. In other words, it is proposed that the low strengths observed were possibly the direct result of taking and testing of cores too early. Some graphs illustrating age-strength relationships for this project are attached. It is felt they help substantiate the above mentioned possibility.

Graph No. 1 consists of a plot of average age versus average strength of mainline core results after they were grouped into categories consisting of all original cores testing above 3500 psi, all original cores testing in the range of 3000 to 3500 psi, all original cores testing below 3000 psi and overall average age and strength.

Graph No. II consists of a plot of average age versus average strength of cores divided into categories by age groups only. The age groups were chosen to obtain six individual points and as nearly as possible the same number of results in each to avoid plotted points being biased by abnormally high or low individual results. The attachments to the graphs illustrate the basis on which they were plotted.

Other data which is independent of that used in plotting the above mentioned graphs, but which also shows evidence that the concrete was still gaining strength at the time the original cores were tested is found on sheet 6 of 6 of the attached core drill report. This compares the strength of the original cores testing below 3000 psi with subsequent check cores taken from very near the original core locations. Fourteen of the fifteen cores gained strength on re-coring and testing. Also there was an overall average strength gain of 600 psi possibly due to the extra aging of the concrete.

It is noted that both graphs indicate strength gains with increasing age. Also from graph No. I it appears that an average strength of 4265 psi and average age of 43 days was necessary to insure no individual results falling below 3500 psi. Similar inspection of graph No. II shows that actual average strengths at 43 days of age were approximately 4300 psi. This appears to indicate that, for this project, testing of all cores should have been delayed until the concrete was approximately 43 days old if the probability of failures were to be minimized. Although 43 days is considerably longer than the 28 day criteria allowed for cylinders it is felt that field curing conditions for the pavement itself are a compromise to that which is or should be given a 28 day standard cylinder. Also some concrete authorities indicate that cores should inherently test lower than standard cylinders at a given age, due to damage resulting from drilling, even with identical curing conditions provided. This suggests that concrete represented by cores should be allowed to age at least 28 days and probably more if results are expected to always be comparable to 28 day cylinder strengths or meet the same minimum strength requirements. Certainly there may be times when core strengths would exceed specified minimum strength levels even at very early ages, but this might require that ambient conditions, construction practices, potential strength of the mix and damage done to cores in drilling (all of which are variable) be favorable.

It is felt that core strengths on this project would have been satisfactory and the incidence of failures low had more time been allowed for the concrete to gain strength. For this reason, I recommend that further investigation be either foregone for this project or at least held to a minimum due in part to the danger and inconvenience to the public which would result from extensive on-the-job testing which may be undertaken. If, however, further investigations are mandatory it is recommended that they begin by the taking of not more than five or ten more cores at selected locations to verify if the concrete strength is now at a generally acceptable level. If found to be acceptable, further investigation could be terminated.

It is further recommended that, in the future and if more than an academic interest is to be taken in the results of core strengths, age ranges or minimum ages at which cores will be taken and tested should be established. Arbitrarily chosen ages, perhaps somewhat greater than 28 days, would be preferable to having no definite policy and would provide a basis for establishing the percentage of failures that would be tolerable. (As it is often expedient to take cores as early as possible for depth measurements, an alternative to the above would be to take due consideration of cores which might be taken and/or tested too early to obtain satisfactory strength by applying an appropriate strength reduction).

As a matter of further information it is noted that Mr. Seamann of the Bureau of Public Roads indicated in his letter of February 27 to Mr. Neiser, that although core strengths do not form a basis of job acceptance not more than 10 percent of cores testing less than the minimum expected 28 day cylinder strength of 3500 psi would be normally expected. This implies that further investigation would be in order if low core breaks do exceed 10 percent. However, it seems that a definition of the ages at which cores would be taken and tested should be added prior to establishing policies that would require further investigation of core strengths.

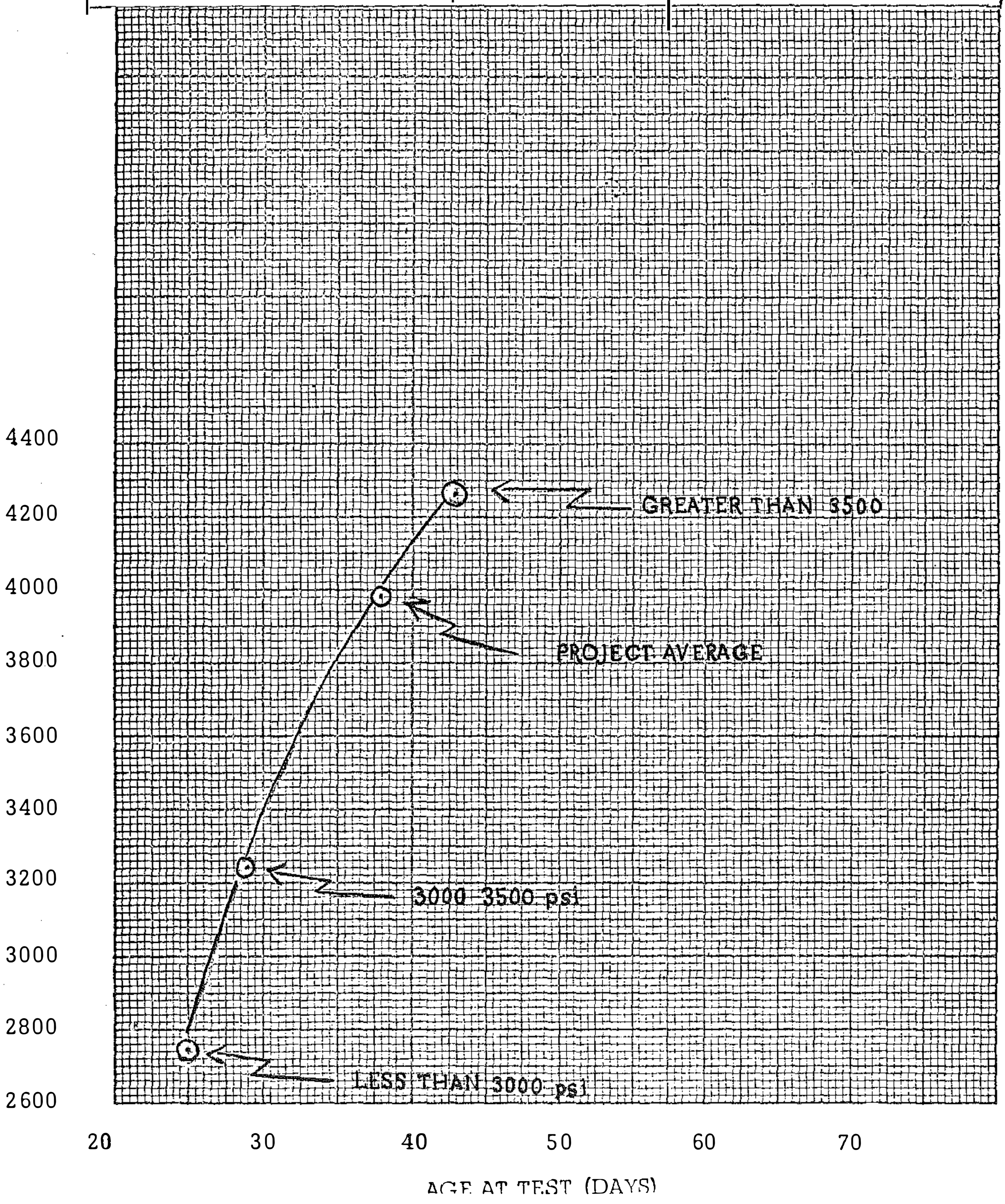
If you have any questions, please advise.

JMC:WHB:lw
Attachments

GRAPH NO. 1

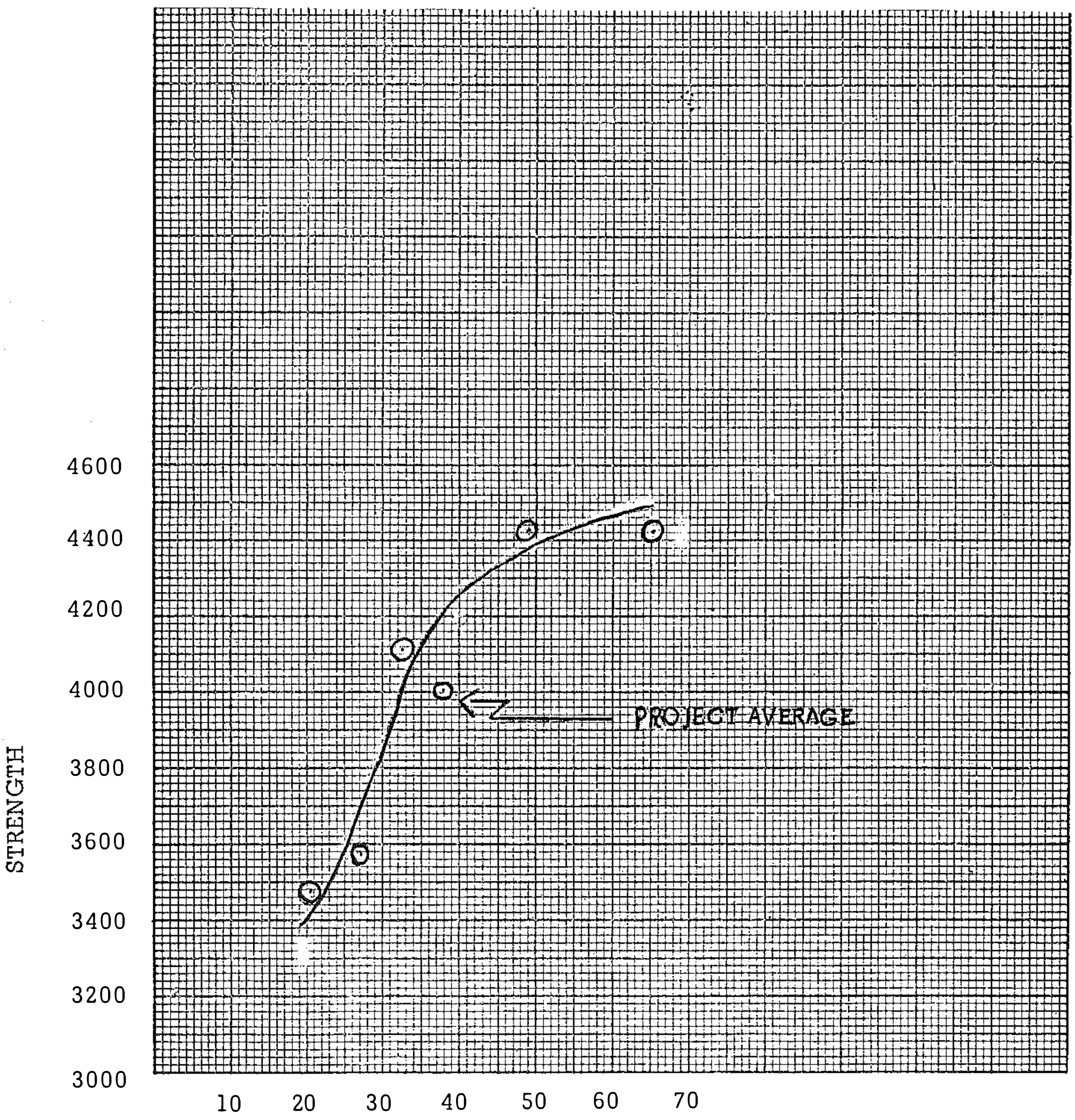
MAINLINE - ORIGINAL CORES ONLY

STRENGTH RANGE	AVERAGE STR.	AVERAGE AGE
Cores Less Than 3000 psi	2738	25 days
Greater Than 3000, Less than 35000	3246	29 days
Cores Greater Than 3500 psi	4265	43 days
Overall Project Average	3990	38 days



GRAPH NO. 2
MAINLINE - ORIGINAL CORES ONLY

Number Cores	Age Range	Average Age	Average Str.
39	Less Than 24 Days	20.3	3462
38	24 Thru 28 Days Inclusive	26.8	3573
38	29 Thru 34 Days Inclusive	32.3	4103
39	35 Thru 43 Days Inclusive	39.7	4196
32	44 Thru 56 Days Inclusive	48.4	4423
32	Older Than 56 Days	64.1	4420
220			



DIVISION OF MATERIALS
KENTUCKY DEPARTMENT OF HIGHWAYS
Frankfort, Kentucky

PAVEMENT CORE DRILL REPORT

COUNTY Bath - Montgomery

PROJECT I 64-6 (7) 109 SP 6-404-22C1 SP 87-557-25C1

Road Name Louisville-Lexington-Catlettsburg

Sheet 1 of 6

Date 3-20 19 68

Lab. No.'s 25006 - 067

Design Thickness 10"

CORE NO.	STATION NO.	STRENGTH AGE	STRENGTH P.S.I.	PAVEMENT DEPTH	CORE NO.	STATION NO.	STRENGTH AGE	STRENGTH P.S.I.	PAVEMENT DEPTH
	EAST BOUND - RIGHT					EAST BOUND - LEFT			
1	1219+12	37	4475	10.35	56	1216+46	37	3970	10.40
2	1228+22	35	4475	10.20	57	1226+04	35	4620	10.30
3	1239+07	34	4690	10.20	58	1235+98	34	4115	10.10
4	1250+08	112	5775	10.25	59	1245+95	34	4330	10.30
5	1258+71	33	4980	10.40	60	1256+17	33	4620	10.10
6	1269+17	33	4690	10.15	61	1266+76	33	4765	10.20
7	1280+06	33	4620	10.30	62	1276+67	33	3970	10.45
8	1289+79	32	4260	10.25	63	1286+02	32	4260	10.40
9	1300+05	32	4475	10.15	64	1296+30	32	4690	10.30
10	1309+62	31	4185	10.60	65	1306+15	31	4115	9.90
11	1320+33	31	4765	10.35	66	1316+30	31	4475	10.40
12	1329+52	28	3680	10.30	67	1325+95	28	4980	10.20
13	1340+02	28	4545	10.50	68	1336+08	28	4260	10.65
14	1350+00	28	3900	10.30	69	1346+14	28	4115	10.25
15	1360+00	27	4475	10.25	70	1356+71	27	4475	10.35
16	1369+55	27	4980	10.30	71	1366+73	27	4390	10.20
17	1379+57	27	4910	10.40	72	1376+00	27	4620	10.30
18	1389+21	20	3250	10.05	73	1386+14	20	3535	10.30
19	1399+69	20	4080	10.30	74	1396+71	20	4115	10.30
20	1409+94	19	4620	10.35	75	1406+85	20	4690	(a) 9.90
21	1419+75	18	3535	10.20	76	1416+10	(b)	(b)	(b)
22	1429+80	67	3755	9.90	77	1426+00	67	4190	10.20
23	1438+80	67	3900	10.10	78	1436+10	67	3900	9.80
24	1449+00	66	5050	10.10	79	1446+04	66	4690	10.25
25	1459+21	66	4690	10.40	80	1456+20	66	4840	10.25
26	1469+09	65	4190	10.25	81	1466+33	65	3940	10.50
27	1479+09	64	4690	10.20	82	1476+12	65	4190	10.70
28	1488+99	64	3540	10.10	83	1486+33	63	4475	10.10
29	1499+42	63	4260	10.55	84	1496+25	63	4475	9.90
30	1511+13	62	4330	10.10	85	1508+04	62	4405	10.70
31	1521+21	61	4765	10.20	86	1518+21	61	3825	10.00

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See Notes Page 5 of 6 & 6 of 6

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PAVEMENT CORE DRILL REPORT

COUNTY Bath - Montgomery | Sheet 2 of 6

PROJECT I 64-6 (7) 109 | Date 3-20 19 68

Road Name Louisville-Lexington-Catlettsburg | Lab. No.'s 25068 - 119

Design Thickness 10"

CORE NO.	STATION NO.	STRENGTH AGE	STRENGTH P.S.I.	PAVEMENT DEPTH	CORE NO.	STATION NO.	STRENGTH AGE	STRENGTH P.S.I.	PAVEMENT DEPTH
	EAST BOUND - RIGHT con't					EAST BOUND - LEFT con't			
32	1531+00	61	4115	9.80	87	1528+29	61	4620	10.10
33	1544+06	60	4330	10.00	88	1541+05	60	4330	9.80
34	1554+19	59	4840	9.90	89	1551+06	60	4810	10.50
35	1564+21	57	3970	10.20	90	1561+05	59	4260	9.90
36	1574+06	57	5200	10.50	91	1571+65	57	4690	10.00
37	1584+12	56	4405	10.20	92	1581+33	57	4405	10.25
[illegible]	1594+04	56	4620	10.40	93	1591+27	56	4115	9.90
39	1604+12	(c)	(c)	(c)	94	1601+25	55	4260	10.10
40	1614+04	55	4330	10.20	95	1611+27	52	4115	10.60
41	1627+78	50	4190	10.10	96	1624+24	50	4690	10.60
42	1637+90	48	4620	10.10	97	1635+35	50	4330	10.35
43	1647+77	48	3755	10.60	98	1644+70	48	4260	10.30
44	1657+95	47	4910	10.35	99	1654+91	48	4690	10.25
45	1667+82	47	4620	9.95	100	1664+70	47	5200	10.30
46	1677+88	46	4910	10.25	101	1674+79	47	4765	9.80
47	1687+94	46	3395	10.10	102	1684+89	46	4620	10.05
48	1697+92	45	4190	10.60	103	1694+91	45	4475	10.40
49	1707+99	45	4115	9.95	104	1704+05	45	4260	10.25
50	1720+70	40	4765	10.55	105	1717+87	40	4405	9.90
51	1730+64	40	4045	10.60	106	1727+59	40	3825	10.25
52	1740+91	39	3610	10.40	107	1737+85	39	4620	10.00
53	1750+85	39	3900	10.30	108	1747+42	39	4690	10.25
54	1761+00	39	4840	10.00	109	1757+79	39	4620	9.95
55	1770+51	32	4260	10.10	110	1767+49	32	3755	10.45
	RAMP B	US 60	Ewington)			RAMP A	US 60	Ewington)	
111	2+60	47	4040	10.70	113	14+50	45	4040	10.70
112	12+70	50	4186	10.90	114	23+90	45	3250	10.00

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PAVEMENT CORE DRILL REPORT

COUNTY Bath - Montgomery

PROJECT I 64-6 (7) 109 SP 4-606-22C1 SP 87-557-25C1

Road Name Louisville-Lexington-Catlettsburg

Sheet 3 of 6

Date 3-20 19 68

Lab. No.'s 25120 - 181

Design Thickness 10"

CORE NO.	STATION NO.	STRENGTH AGE	STRENGTH P.S.I.	PAVEMENT DEPTH	CORE NO.	STATION NO.	STRENGTH AGE	STRENGTH P.S.I.	PAVEMENT DEPTH
	WEST BOUND - RIGHT					WEST BOUND - LEFT			
115	1217+06	18	*2743	10.50	170	1219+70	18	3320	10.15
116	1227+73	18	3176	10.40	171	1231+21	18	3501	10.20
117	1238+17	19	*2815	10.40	172	1241+20	19	3573	10.10
118	1248+00	19	*2887	10.30	173	1250+40	19	3465	10.20
119	1259+00	19	3320	10.00	174	1260+85	19	3753	10.20
120	1268+74	19	*2671	10.20	175	1271+27	19	3031	10.10
[illegible]	1277+73	20	3970	10.45	176	1281+42	20	3609	10.50
[illegible]	1287+67	20	4006	10.00	177	1291+25	20	4150	10.25
123	1298+07	20	3537	10.60	178	1301+33	20	3970	10.15
124	1308+57	20	*2526	10.55	179	1311+66	22	3681	10.25
125	1318+88	22	3248	10.50	180	1321+36	22	3573	10.10
126	1327+30	22	3825	10.15	181	1331+45	22	3140	10.20
127	1337+80	22	3248	10.30	182	1340+89	22	3104	10.10
128	1348+39	22	*2815	10.15	183	1350+91	23	*2851	10.20
129	1357+02	23	3753	10.30	184	1361+24	23	3248	10.40
130	1367+50	23	3392	10.20	185	1371+06	23	3320	10.25
131	1378+00	25	3356	10.40	186	1381+36	25	3248	10.25
132	1388+39	25	3176	10.35	187	1391+27	25	*2959	10.20
133	1397+42	25	*2851	10.30	188	1401+16	25	*2562	9.85
134	1407+85	25	3176	10.35	189	1410+22	26	3645	10.30
135	1418+47	26	3898	10.25	190	1421+08	26	3681	10.30
136	1428+10	26	3140	10.15	191	1431+01	26	3032	10.20
137	1438+22	26	*2454	10.60	192	1441+19	26	3248	10.10
138	1448+03	27	*2959	10.20	193	1450+98	27	3573	10.10
139	1458+16	27	*2743	10.30	194	1461+48	27	*2815	10.10
140	1467+90	27	3248	9.90	195	1471+65	27	3322	10.10
141	1477+97	28	3212	10.65	196	1481+07	28	3680	(d) 9.95
142	1488+13	28	3248	10.20	197	1490+75	28	3753	10.10
143	1498+03	28	3104	10.45	198	1500+54	28	*2382	10.10
144	1511+15	29	3645	10.20	199	1514+06	29	4186	10.25
145	1521+00	29	4547	10.15	200	1524+03	29	3212	10.20

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Harold ... (signature)

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PAVEMENT CORE DRILL REPORT

COUNTY Bath - Montgomery

PROJECT I 64-6 (7) 109 SP 6-404-22C1 SP 87-557-25C1

Road Name Louisville-Lexington-Catlettsburg

Sheet 4 of 6

Date 3-20 19 68

Lab. No.'s 25182 - 233

Design Thickness 10"

CORE NO.	STATION NO.	STRENGTH AGE	STRENGTH P.S.I.	PAVEMENT DEPTH	CORE NO.	STATION NO.	STRENGTH AGE	STRENGTH P.S.I.	PAVEMENT DEPTH
	WEST BOUND RIGHT - (Con't					WEST BOUND - LEFT con't			
146	1530+91	32	4331	10.00	201	1537+05	32	3392	10.30
147	1544+40	32	4042	10.30	202	1547+00	33	3537	10.30
148	1554+20	33	3609	10.00	203	1556+95	33	4042	10.20
149	1564+30	33	4403	10.25	204	1566+94	33	3392	10.30
150	1573+93	33	3898	10.10	205	1577+02	33	3104	10.50
151	1583+88	34	3753	10.10	206	1587+03	34	3176	10.50
152	1594+76	34	3825	10.20	207	1597+00	34	3465	10.30
153	1604+24	34	4331	10.30	208	1607+10	36	3609	10.20
154	1614+03	36	4042	10.30	209	1617+08	36	4403	10.10
155	1627+42	38	3609	10.35	210	1630+00	38	4150	10.50
156	1637+33	39	4403	10.60	211	1639+76	39	4331	10.20
157	1647+69	39	4547	10.25	212	1649+75	39	4186	10.10
158	1657+24	39	4475	10.20	213	1659+79	39	3681	10.25
159	1667+01	40	4620	9.80	214	1670+02	40	3970	10.30
160	1677+06	41	3609	10.20	215	1679+76	41	4186	10.05
161	1686+12	43	3212	10.25	216	1690+00	43	4042	9.95
162	1696+61	43	3970	10.10	217	1699+88	43	4331	10.00
163	1706+79	43	*2526	10.00	218	1710+03	43	4042	9.85
164	1720+00	43	4836	10.20	219	1723+00	43	5053	10.00
165	1730+18	43	4403	10.00	220	1733+24	43	4547	9.90
166	1740+00	46	5486	10.20	221	1743+21	46	4439	9.90
167	1750+35	46	4259	10.00	222	1753+04	46	5125	10.00
168	1760+25	46	4042	10.10	223	1763+34	46	3934	10.10
169	1770+04	47	4728	10.00	224	1773+10	47	3680	10.20
	RAMP A (US 60 EAST)					RAMP D (US 60 EAST)			
225	2+00	72	3753	10.00	226	10+80	75	3680	10.00
226	16+45	71	3465	10.10	227	24+40	78	4620	10.70

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Howard Lena M. [signature]

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PAVEMENT CORE DRILL REPORT

COUNTY Bath - Montgomery

PROJECT I 64-6 (7) 109 SP 6-404-22C1 SP 87-557-25C1

Road Name Louisville-Lexington-Catlettsburg

Sheet 5 of 6

Date 3-20 19 68

Lab. No.'s 25234 - 248

Design Thickness 10"

CORE NO.	STATION NO.	STRENGTH AGE	STRENGTH P.S.I.	PAVEMENT DEPTH	CORE NO.	STATION NO.	STRENGTH AGE	STRENGTH P.S.I.	PAVEMENT DEPTH
				KY 36 INTERCHANGE					
	RAMP A					RAMP B			
228	3+65	64	4260	10.40	230	3+00	58	3890	11.10
229	16+45	61	4620	10.40	231	15+55	56	4510	10.30
	RAMP C								
232	1+35	56	3680	10.50		RAMP D			
233	1+50	68	4150	10.40	235	1+90	68	3320	10.10
[illegible]	16+35	64	4400	10.20	236	15+00	70	3535	10.40
	INCIDENTAL PAVING								
237	1670+34	64	4583	10.40 (RAMP WEDGE)					
238	9+80	77	3970	11.30 (RAMP D RADIUM US 60)					
	NOTES								
75B	1403+85			10.20	196B	1478+07			10.30
75	1406+85			9.40	196	1481+07			9.60
75A	1409+85			10.10	196A	1484+07			10.00
	(a) Average Unit # 75			9.90		(d) Average Unit 196			9.93

(b) Approximately 70' of Pavement Removed after drilling was completed.
Original core height = 10.00" Strength 6065 PSI

(c) Approximately 195' of Pavement Removed after drilling was completed.
Original core height = 10.30 Strength 4620 PSI

* Original Core Strength below 3000 PSI and Rechecked - See Page 6 of 6

Average Placement of Mesh Depth = 4.46"

Actual Average Pavement Thickness = 10.33"

Avg Str 3990 psi

Cyls

Copies to:
16 Avg Age Cyls < 3000 psi - 24.6 days
39 Ave Age Cyls < 3500 psi - 27.6 days
183 Ave Age Cyls > 3500 psi - 42.7 days
Total 237 Total 39.5 days Includes Ramps

Cyls greater than 3500 are 55% older than those under 3500 psi.

Harold [illegible]

Director of Materials

abp

DIVISION OF MATERIALS
KENTUCKY DEPARTMENT OF HIGHWAYS
Frankfort, Kentucky

PAVEMENT CORE DRILL REPORT

COUNTY Bath - Montgomery   Sheet 6 of 6

PROJECT I 64-6 (7) 109 SP 6-404-22C1 SP 87-557-25C1   Date 3-20 19 68

Road Name Louisville-Lexington-Catlettsburg   Lab. No.'s 25249 - 56

Design Thickness 10"

CORE NO.	STATION NO.	Original Core PSI	Age	Increase in PSI	Check Core PSI	Age	
	WEST BOUND - RIGHT						
*	1217+06	2743	18	26%	3465	52	
*	1238+17	2815	19	18%	3320	53	
*	1248+00	2887	19	-2.6%	2815-2850 ?	53	The only one that failed to gain
*	1268+64	2671	19	32%	3535	53	
	1348+39	2815	22	14%	3210	56	
	1397+42	2851	25	18%	3355	59	
*	1438+22	2454	26	53%	3750	60	
*	1448+03	2960	27	24%	3680	61	
*	1458+16	2743	27	21%	3320	61	
*	1706+79	2562	43	66%	4260	? 136	?
	WEST BOUND - LEFT						
*	1350+91	2851	23	11%	3175	57	
*	1391+57	2959	25	12%	3320	59	
*	1401+16	2562	25	38%	3535	59	
*	1461+48	2815	27	15%	3250	61	
*	1500+54	2382	28	8%	2565	62	
	Average	2738	24.9	22%	3338	62.2	

NOTE (1) All units meet the specified tolerances, as set forth in the 1965 Kentucky Highway Specifications Section 307.3.18

NOTE (2) Additional Core information - See Report dated 8/28/67 Lab. No. 15036 thru 50

This concrete apparently gained an average of 600 psi by ~~[illegible]~~ aging 37 additional days after being tested at 25 days.

Harold [illegible] Mays